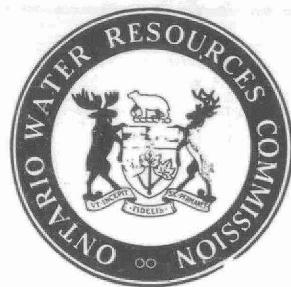


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WASTE STABILIZATION POND

BERM EROSION

THE ONTARIO WATER RESOURCES COMMISSION

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REPORT
ON
WASTE STABILIZATION POND
BERM EROSION

BY:
M.B. FIELDING
DIVISION OF RESEARCH
ONTARIO WATER RESOURCES COMMISSION

JUNE, 1966

TABLE OF CONTENTS

LIST OF FIGURES.....	ii
SUMMARY.....	1
1.0 INTRODUCTION.....	2
2.0 WAVE THEORY.....	2
3.0 WAVE HEIGHT PREDICTION.....	7
3.1 General.....	7
3.2 Wind Velocity.....	7
3.3 Effective Fetch.....	7
3.4 Significant Wave Height.....	8
4.0 BERM CONFIGURATION.....	11
4.1 Existing Berms.....	11
4.2 Field Testing.....	11
4.3 Laboratory Testing.....	15
4.3.1 Wave Tank Construction.....	15
4.3.2 Test Procedure.....	15
4.3.3 Test Results.....	18
4.4 Interpretation of Test Results.....	18
5.0 ENERGY DISSIPATION AND SHORE PROTECTION.....	20
5.1 Energy Dissipators.....	22
5.2 Shore Protection.....	22
5.2.1 Rip-rap.....	22
5.2.2 Soil-cement.....	23
5.2.3 Sand bagging.....	23
5.3 General.....	24
5.3.1 Plastic film.....	24
5.3.2 Snow fencing.....	24
5.3.3 Asphalt membrane.....	24
6.0 SUGGESTED BERM DESIGN.....	25
6.1 General.....	25
6.2 Assumed Conditions.....	25
6.3 Calculation.....	25

7.0 CONCLUSIONS.....	32
8.0 RECOMMENDATIONS.....	32
APPENDIX-A.....	34
APPENDIX-B.....	39
REFERENCES AND BIBLIOGRAPHY.....	41

LIST OF FIGURES

<u>Figure</u>		<u>Page No.</u>
1	Wave Height-Fetch-Wind Velocity Correlation.....	5
2	Windbreak Effect.....	9
3	Effective Fetch.....	10
4	Frequency Distribution of Wave Height Within any particular Wave Train.....	12
5	Standard Berm Cross-Section.....	13
6	Wave Gauging Staff.....	14
7	Laboratory Wave Tank.....	16
8	Graph of Erosion Rate vs Embankment Slope..	19
9	Berm Slope - Grain Size - Correlation.....	21
10	Graph of Pond Area vs Fetch.....	26
10A	Graph of Pond Area vs Fetch.....	27
11	Calculation of Significant Wave Height.....	28
11A	Calculation of Significant Wave Height.....	29
12	Graph of Wave Run-up vs Embankment Slope...	30

SUMMARY

In the Province of Ontario waste stabilization ponds are a relatively common method of sewage treatment for small municipalities.

The erosion of the cell berms by wave action has been a common, and in some cases, severe, problem in these ponds. The report presented herewith, prepared on the basis of investigations both in the field and in the laboratory, and from information gained from published material, examines the causes of erosion, provides means of predicting erosion rates and effects, and provides methods of reducing or eliminating erosion of waste stabilization pond berms by wave action.

From the investigations made it is concluded that the rate of berm erosion is dependant not only upon the height of wave impinging on the berm but to an even greater extent upon the inter-relationship of the berm material and slope. For any berm material, as classified by median grain size, there is a slope which will be stable under wave action.

Information is provided so that this stable slope may be selected on the basis of a sieve analysis of the berm material.

Information is also provided so that wave heights and therefore the required freeboard may be calculated.

In some cases the available berm material will be such that the construction of a naturally stable slope will not be economical, and therefore some form of energy dissipation, and/or shore protection will be required. Floating log booms and sand bagging at the water-line have both proved effective in this respect and other methods are available.

1.0 Introduction:

In Ontario the use of waste stabilization ponds for the treatment of both domestic and industrial wastes was initiated in 1956, when an installation was built to serve the army camp at Ipperwash. The system has received wide acceptance and there are now approximately 35 waste stabilization ponds in Ontario, with cell sizes ranging from 2 to 42 acres.

The recommended inside berm slope on all installations to date has been 1:4 (vertical:horizontal). Actual installations have had berm slopes ranging from 1:2 to 1:6 with the average being approximately the 1:4 recommended. As the installations have aged it has become increasingly apparent that erosion of the berms, caused chiefly by wave action, is a problem of serious magnitude. The severity of erosion has varied, and from a summary prepared in 1964⁽¹⁾ does not seem to be dependant upon cell size. In several cases erosion proceeded, in a period of two years, to the point where major repairs were required.

It is this prevalence of erosion damage of waste stabilization pond berms which has prompted the preparation of the report presented herewith.

The intention of this report is to present information regarding wave formation and action, effects of waves on berms of different materials, and the efficiency of energy dissipation devices. Recommendations for design and construction practices which will minimize the severity of erosion will be presented.

2.0 Wave Theory

The prediction of wave heights which may occur at any given waste stabilization pond installation is of importance for two reasons. The prime reason is for the calculation of the required freeboard. It is felt

that occasional over topping of the berms (possibly 15% of the waves during the most severe storm experienced) will not be detrimental to the berms, and reduction of berm height would produce an appreciable saving in construction costs. A secondary reason for calculation of maximum wave height and average wave height is to determine the rate of erosion which may be anticipated. Erosion is more rapid under the effects of large waves but the cumulative effect of small waves will be as severe, though spread over a longer period of time. (This assumes a soil type material, without rip-rap or large boulders.)

In 1874, Thomas Stevenson, a Scottish harbour engineer proposed a mathematical formula for the prediction of wave height (2):

$$H = 1.5\sqrt{D} + (2.5 - \sqrt{D}) \quad .1.$$

in which H = Height of wave, feet
D = Fetch, nautical miles

Wind velocity is not included in the above formula but is recognized as being a variable. In the original work Stevenson used steady wind velocity of 78 knots (90 miles per hour).

From the foregoing it is apparent that the Stevenson formula is not applicable to small bodies of water (fetch less than one nautical mile) or for wind velocities other than 90 mph. This formula is presented here as it is the one most commonly encountered in design text books.

Other investigations have been made on waves with a view to predicting wave heights (3,4,5,6,7) and most of the more recent works will provide reasonable correlation. For the purpose of this report the graph

of dimensionless parameters (Fig.1) as developed by Charles L. Bretschneider (6) has been used as it has been found (by limited testing) to be applicable to small installations. This will be discussed further under section 4.2 "Field Testing".

The Bretschneider curve provides a correlation between wind velocity, fetch and significant wave height. For the installations being considered in this report the fetch will not exceed approximately 2000 feet (600 metres) and the sustained wind velocities will not exceed 65 mph (3000 cm/sec). In this range the Bretschneider curve is effectively a straight line of the equation:

$$gH_s/U^2 = 0.0026 (gF/U^2)^{0.48} \quad .2.$$

which may be further reduced to

$$H_s = 4.28 \times 10^{-4} F^{0.48} U^{1.04} \quad .3.$$

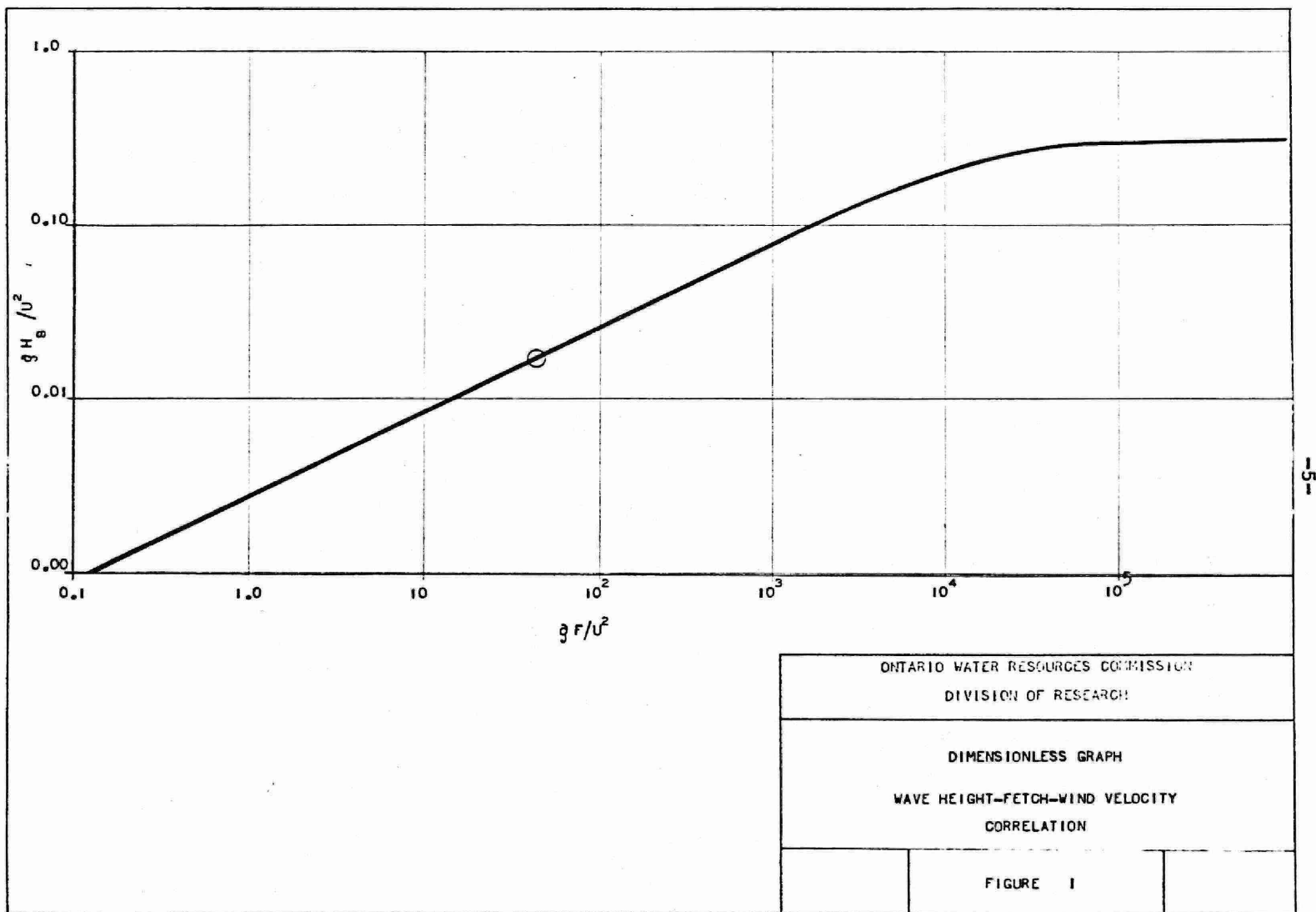
in which

H_s = significant wave height - feet

F = effective fetch - feet

U = wind velocity - feet per second

g = gravitational constant



In metric units the equation becomes:

$$H_s = 7.25 \times 10^{-5} F^{0.48} U^{1.04} \quad .4.$$

in which:

H_s = significant wave height - cm.

F = effective fetch - cm.

U = wind velocity - cm. per second

Wind acting on a body of water not only produces waves but further causes a "piling up" of the water at the downwind boundary of the enclosed water body. This set-up is given by the generally accepted formula: (8)

$$S = \frac{U^2 F}{1400 D} \quad .5.$$

in which

S = set up - feet above stillwater

U = wind velocity - mph

F = fetch - miles

D = water depth (ave.) - feet

Introducing typical values for waste stabilization ponds (ie. $F = 0.2$ miles, $U = 40$ mph $D = 5$ Ft.) the equation yields the following:

$$\begin{aligned} S &= \frac{U^2 F}{1400 D} \\ &= \frac{40^2 \times 0.2}{1400 \times 5} \\ &= 0.046 \text{ feet} \end{aligned}$$

This is not considered significant and therefore wind set up will not be introduced into any further discussions.

3.0 Wave Height Prediction

3.1 General

As discussed in section 2.0, "Wave Theory", the prediction of wave heights will be based on the work done by Bretschneider and using equation 2:

$$gH_s/U^2 = 0.0026 (g^F/U^2)^{0.48}$$

and/or its reduction in either English or Metric units (equations 3 and 4).

3.2 Wind Velocity

Wind velocity records are available for thirty-seven locations in Ontario (9). A search of these records for the past five years provides the following wind velocity data:

max. vel. sustained for one hour	- 65 mph*
" " " " six hours	- 40 ⁺ mph
mean wind vel. (app.)	- 18 mph

* Trenton, March 5, 1964

For the purpose of calculating maximum wave heights it is assumed that a wind duration of less than one hour will not produce the maximum wave height associated with that velocity. Therefore, a maximum wind velocity for calculation of freeboard has been set at 65 mph (3000 cms/sec.).

3.3 Effective Fetch

"Fetch" in the context of this report is defined as the continuous length of water surface over which the wind blows in an essentially constant direction.

A further concept of "effective fetch" is also introduced. This refers to the fetch as defined above but with proper consideration given to the effects of shelter belts, wind breaks and topography.

The most common element which will cause a reduction in fetch is a shelter belt or wind break of trees. Figure 2 illustrates graphically the effect on wind velocities, and thus fetch, of such a shelter belt. (10)

For the purpose of calculating the reduction of fetch caused by a shelter belt, it is assumed that the effect of such shelter belt extends over that distance in which the wind velocity is reduced to less than 80 per cent of its initial velocity. From Fig.2 this distance is taken as 17X shelter belt height. Assuming a shelter belt of height "a" is erected a distance "x" from the upwind shoreline of a body of water having a fetch F_1 . the effective fetch F, becomes:

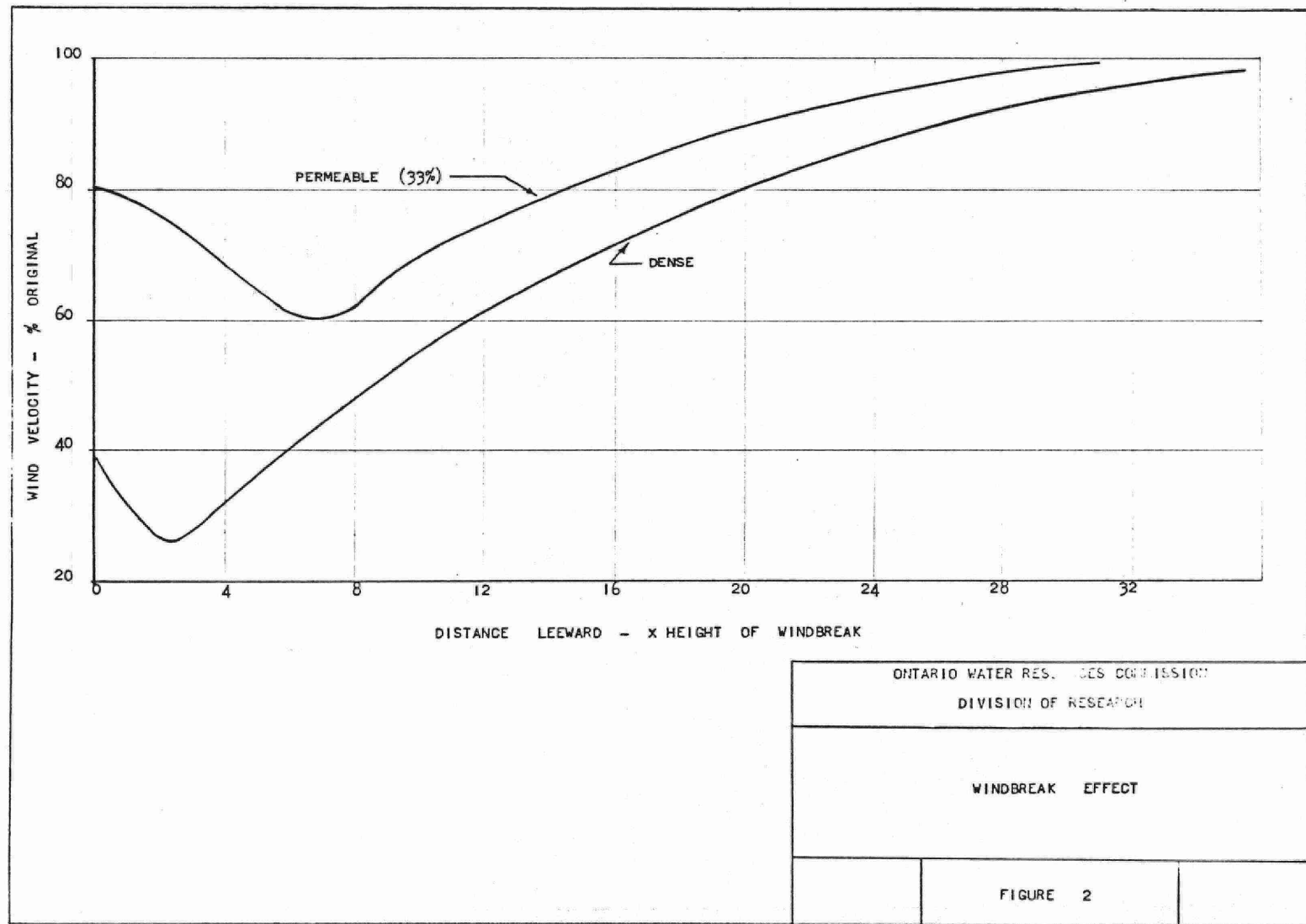
$$F = (F_1 + x) - 17a \quad .6.$$

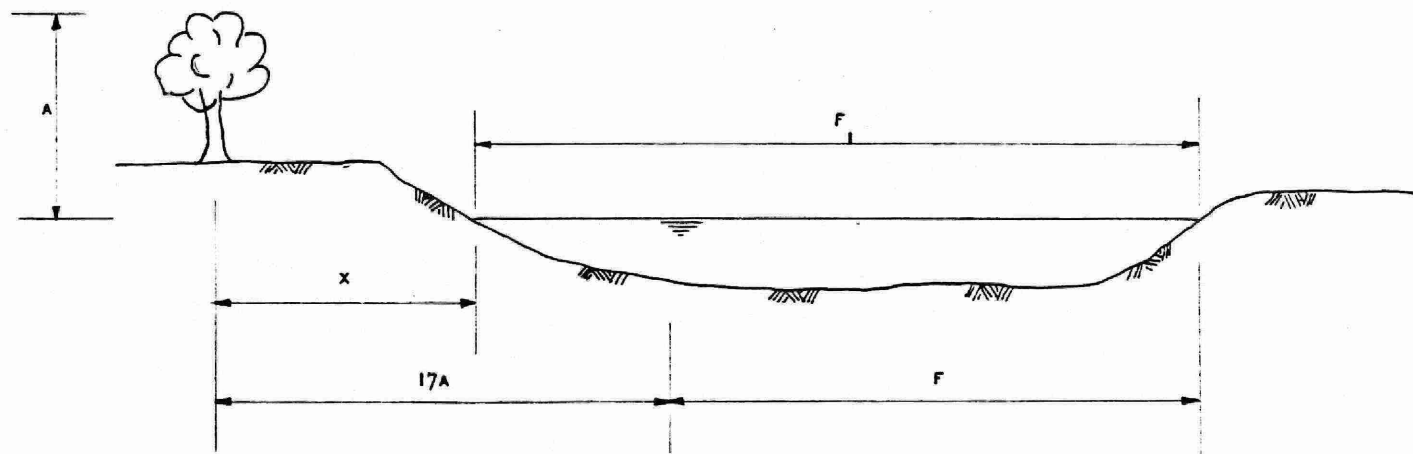
(See Figure 3)

Hereinafter, the effective fetch will be used in all calculations involving fetch.

3.4 Significant Wave Height

The wave height used in calculations to determine freeboard is not the absolute maximum for the particular effective fetch and wind velocity obtained. It is, rather, the significant wave height which is calculated. The significant wave height (H_s) is defined as the average height of the third highest waves obtained for any set of conditions. Figure 4 shows the statistical probability of waves of height greater than H_s occurring in any wave train.





$$F = \text{EFFECTIVE FETCH} = (F_1 + x) - 17A$$

$$(F \text{ MAX.} = F_1)$$

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EFFECTIVE FETCH

NTS

FIGURE 3

In setting a maximum wind velocity for calculation of H_s , a velocity which has occurred once in five years and for a period of one hour, has been used. From Figure 4 it is seen that approximately 85% of the waves occurring at a wind velocity of 65 mph will be equal to, or less than, $H_s(8)$. This data indicates that the use of the criteria proposed will not reduce over-topping completely but that the probability of over-topping is very small. If it is accepted that occasional over-topping is not detrimental (8) it is felt that the criteria are adequate for berm design.

4.0 Berm Configuration

4.1 Existing Berms:

To date (May, 1966) waste stabilization ponds in Ontario have been constructed with berms having a recommended inside slope of 1:4 (vertical:horizontal) although slopes ranging from 1:2 to 1:6 have been used. The typical berm cross-section is illustrated in Figure 5.

Inspection of five existing installations indicated erosion damage, of varying severity, and of a typical nature in all cases. The approximate eroded configuration of the berms is illustrated in Figure 5, by broken lines.

4.2 Field Testing

The accurate evaluation of conditions obtaining in the field was found to be difficult, and the use of field testing procedures for the determination of the effects of waves on berms was not considered to be accurate due to the inability to produce a controlled wave condition.

Wave heights were measured using a gauging staff as shown in Figure 6 and a cine-camera. Wind velocities were determined using a Taylor Wind-Scope. In only

FREQUENCY DISTRIBUTION OF WAVE HEIGHT
WITHIN ANY PARTICULAR WAVE TRAIN

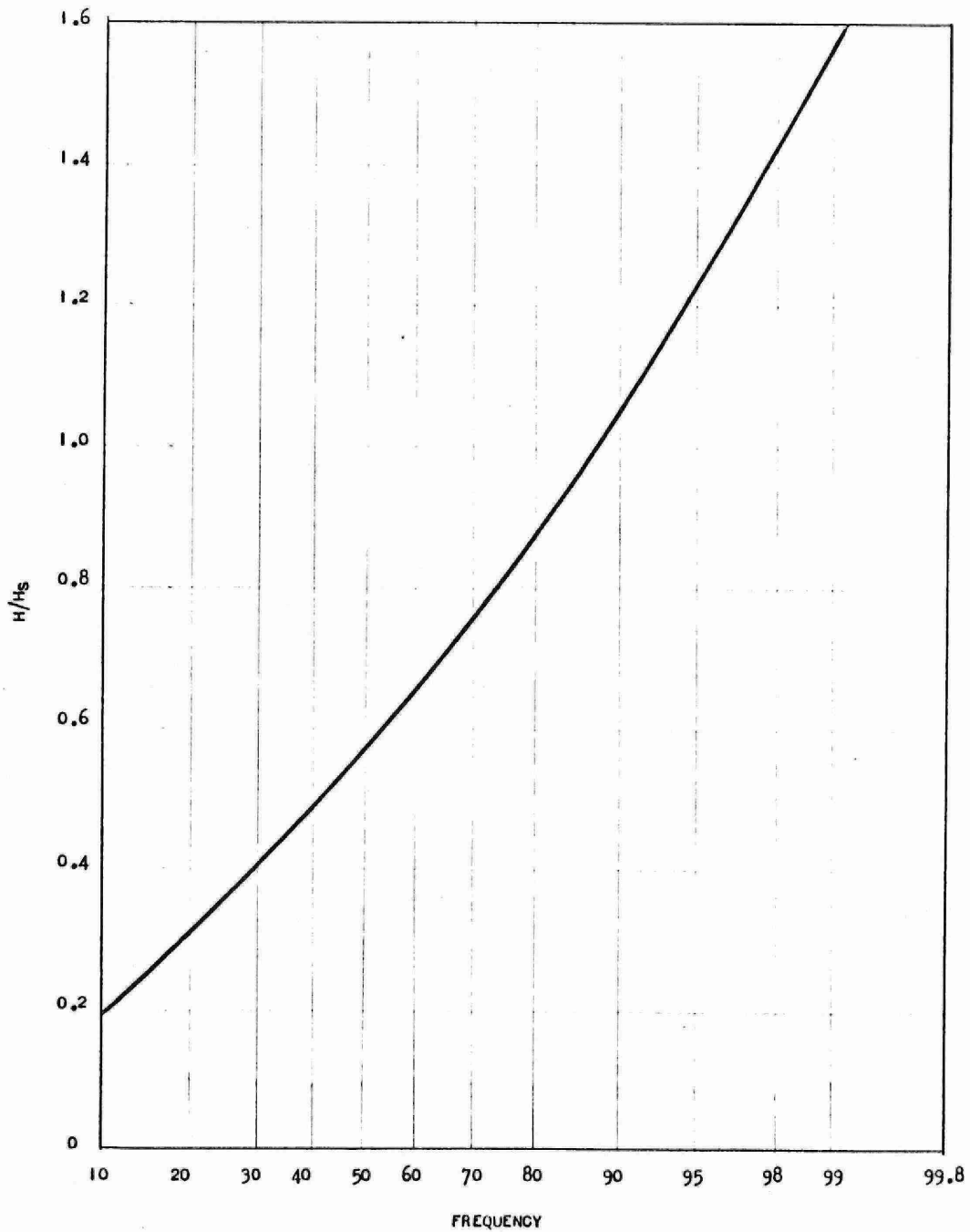
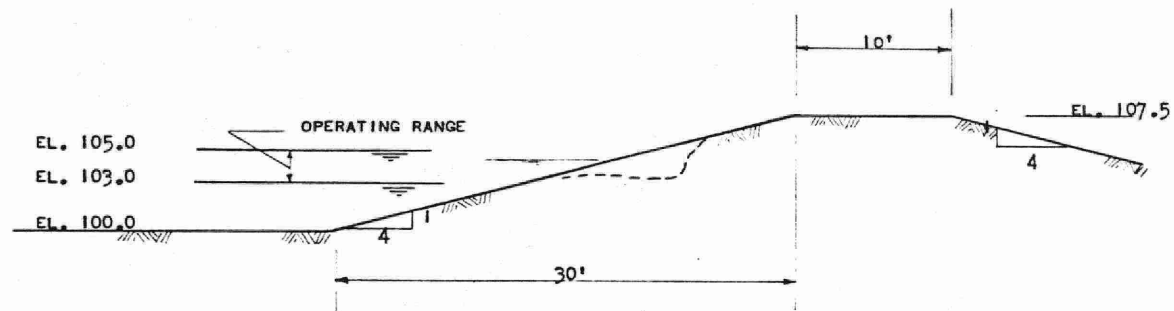


FIGURE 4

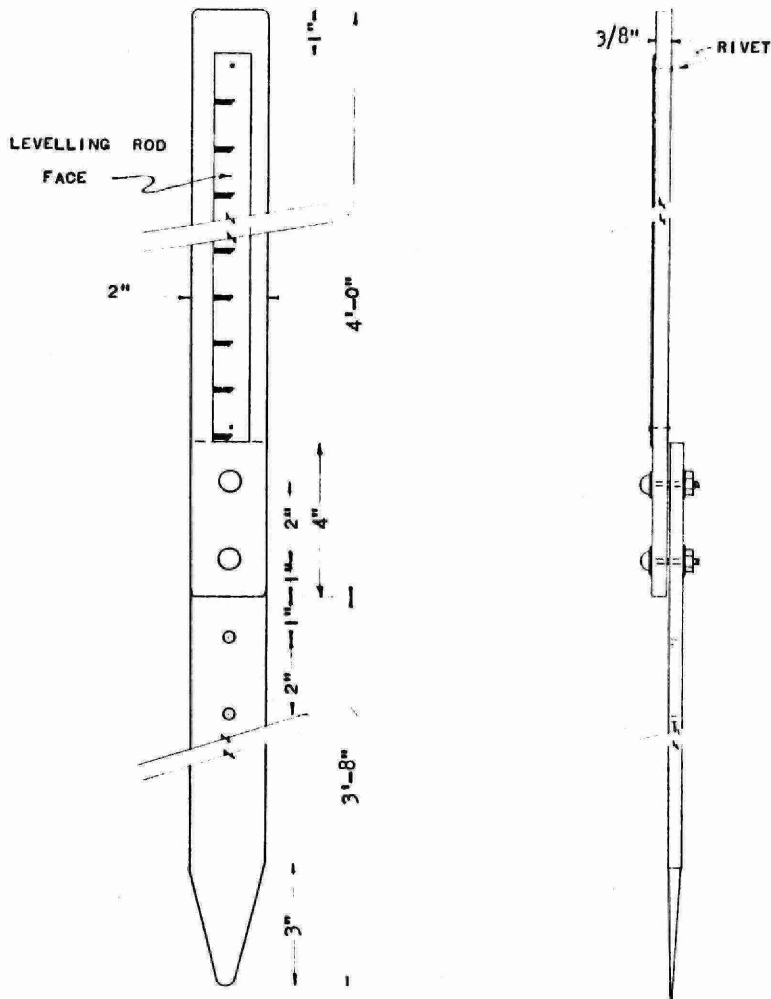


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STANDARD BERM CROSS-SECTION

1"=10'

FIGURE 5



$\frac{1}{4}$ SCALE

WAVE GAUGING STAFF

FIGURE 6

one case was a wind of sufficient velocity (average 20 mph) and duration encountered to give wave build-ups which could be considered reliable. In this instance the correspondence to the Bretschneider curve was exact (see Fig.1). In all other instances the wind velocities and directions were light and variable respectively and were not considered to provide reliable data. The effects of such waves were not measurable over the short period of time they obtained, even though the long term effect was obvious (see Fig.5).

For the reasons outlined above it was decided to construct a "wave tank" in which waves of controlled height and frequency could be produced, and berms of varying materials and slopes constructed. In this tank the effects of waves on the berms were readily observed and measured.

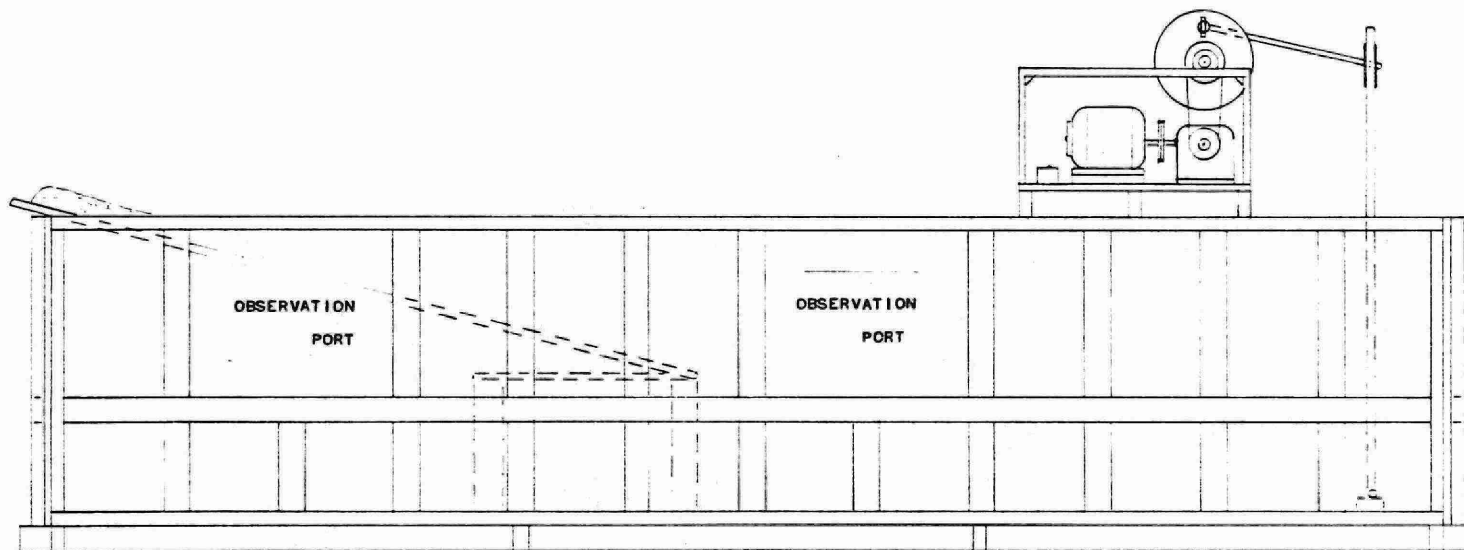
4.3 Laboratory Testing

4.3.1 Wave Tank Construction

A wave tank as shown in Figure 7 was constructed in the OWRC laboratory. The berm slope was infinitely variable from vertical to horizontal and wave height and frequency were variable from zero to 10 inches (25cms) and from 30 to 75 waves per minute respectively. Two observation ports were provided; one near the wave gate to facilitate measuring wave amplitude and the other at the berm, to facilitate observation of the slope reaction to waves.

4.3.2 Test Procedure

The testing of soil samples for resistance to erosion involved setting the inclined berm support at the desired slope and covering it with a uniform, lightly-compacted soil layer to a depth of 5 cms. Every attempt was made to obtain uniform compaction for



-16-

18'-0"

2'-6"

4'-0"

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LABORATORY WAVE TANK

$\frac{3}{8}" = 1'-0"$

FIGURE 7

MF

each test. Two different materials were used (see sieve analysis, appendix B), one of which did not contain any appreciable amount of clay. For this material (sand No.2) the slope was rebuilt after each test with only enough new material being added to make up any losses. The second material (L1) contained clay which was washed into suspension under wave action and therefore a new sample was used for each test.

A line was drawn on the observation port parallel to the soil surface and at a perpendicular distance of 2 cms below the surface line. When any portion of the slope material was eroded to a depth of 2 cms (as indicated by the line) the test was concluded and the time from initiation to this point recorded as Tf.

Waves were produced at a selected amplitude and frequency by adjusting the speed and stroke of the hinged wave gate. It was found as the tests proceeded that an amplitude of 6 cms (trough to crest) and frequency of 30 waves per minute gave a wave action which provided a measurable time of erosion on the steepest slope (1:2) and eroded the shallowest slope (1:6) in a reasonable length of time. Smaller waves would have resulted in some tests extending for an uneconomically long period and larger waves would cause instantaneous wash-out of steeper slopes.

A constant water level in the tank was insured by using an overflow header tank on the wave tank.

Observation of the waves for the purpose of recording amplitude was provided by the port closest to the wave gate. This also provided an opportunity to watch the wave action to insure that a true wave, rather than a clapoti was produced.

Pictures of each test were taken with both a

35 mm still camera and an 8 mm cine-camera.

Any tests in which non-uniform berm conditions, or side effects from the tank walls, produced uneven erosion were aborted and a new test initiated. Such interrupted test were not recorded.

4.3.3 Test Results

The results of the wave tests carried out are given in detail in Appendix A. In general it was found that for any given berm material and wave amplitude the time taken to erode a given depth of material increased as the slope was flattened. The relationship was not linear and as the slope approached the horizontal an inclination was found at which erosion was negligible.

Test results are represented graphically in Figure 8. For purposes of this representation the rate of erosion (g) is taken as K/T_f where K is a constant dependant upon the amount of material eroded in the test and T_f is the time required to produce such erosion.

Grading curves based on sieve analysis of the materials used are given in Appendix B.

4.4 Interpretation of Test Results

From tests conducted in the OWRC laboratory the following observations and conclusions are made:

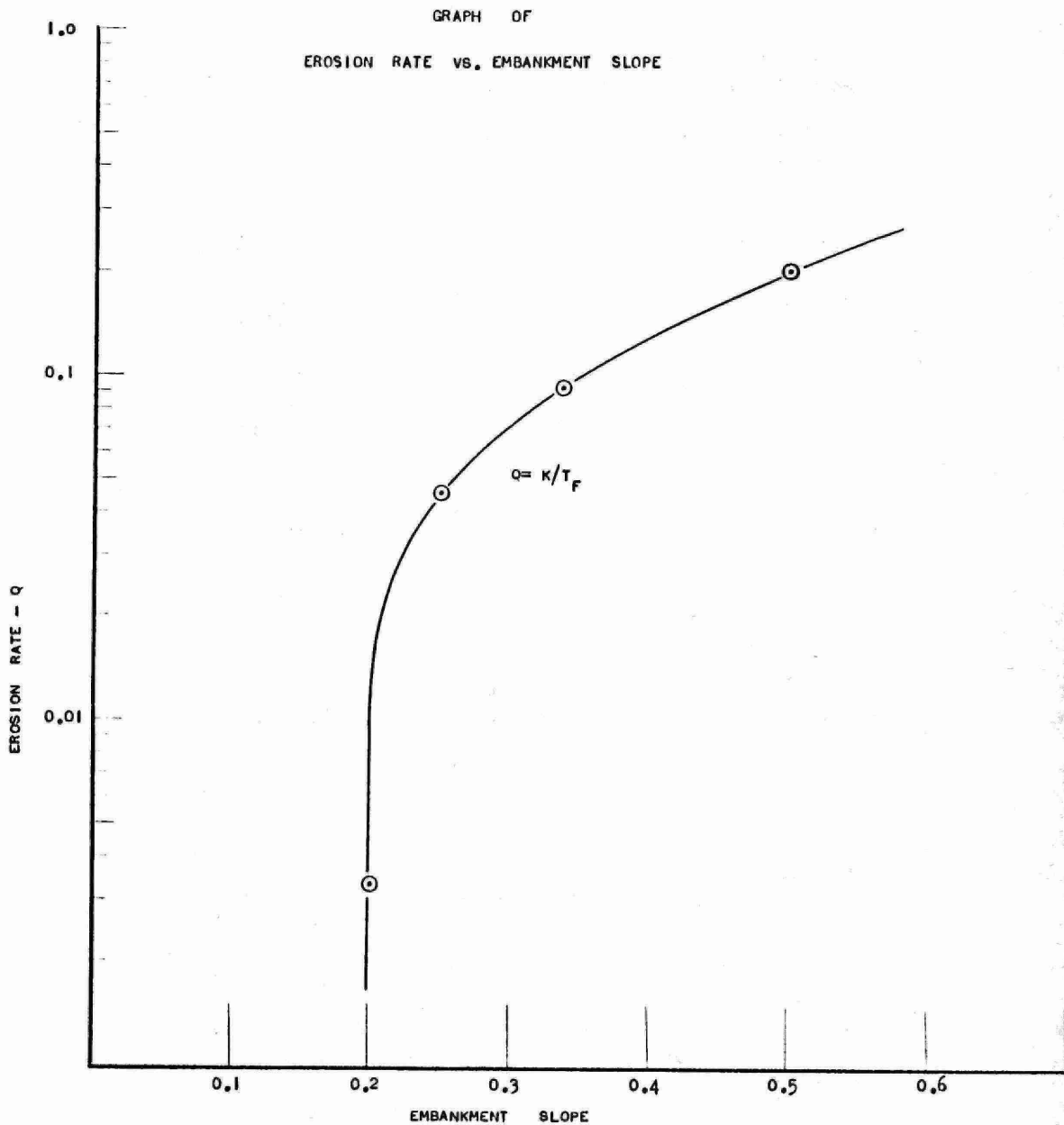


FIGURE 8

1. The rate of erosion of soil berms varies with the amplitude of the waves impinging on the slope.
2. For any given berm material and wave height the erosion rate varies with the berm slope, being more severe on steeper slopes.
3. For any given berm material there is a berm slope at which wave erosion is negligible, such slope being dependant upon the mean grain size and uniformity coefficient of the berm material.

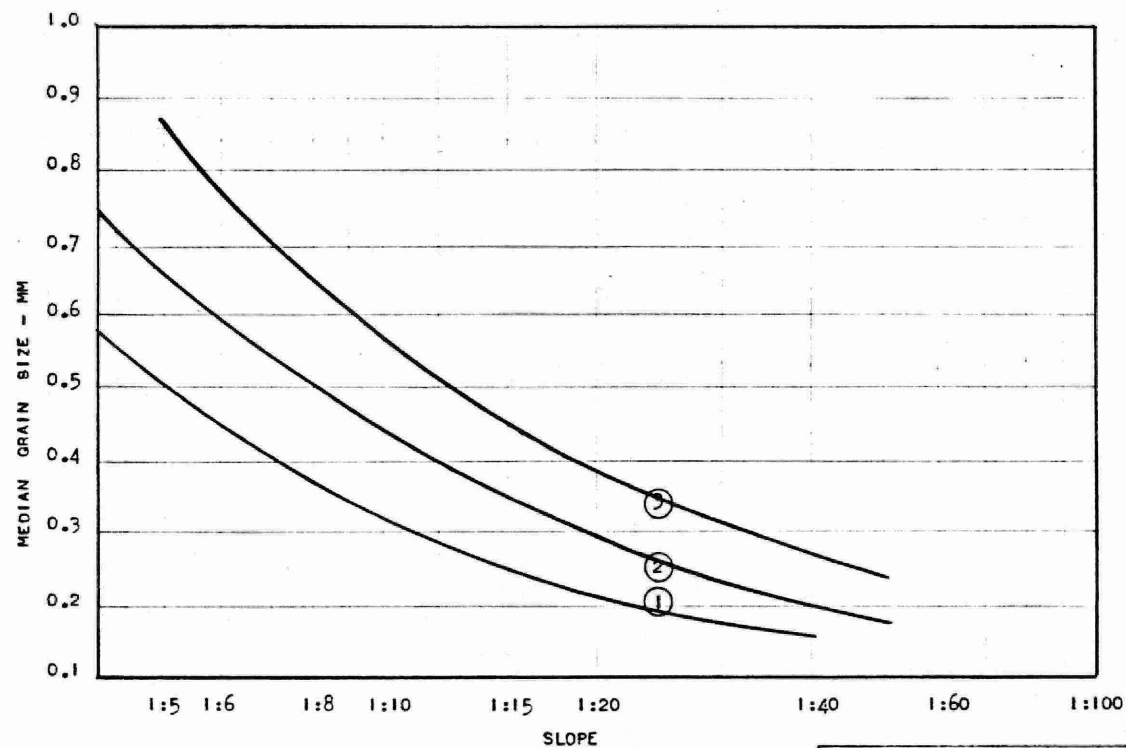
The results of the laboratory tests show close correlation with previous results published by Bagnold (11) and Wiegel (12) who were experimenting with somewhat different objectives.

A graph showing stable slopes related to mean grain size of slope material, as presented by Wiegel is given in Figure 9.

For the purpose of waste stabilization pond berms a protected slope is considered to obtain on ponds of less than 20 acres (L:W 2:1 or less), moderately protected and exposed slopes obtain on ponds of less than 100 acres and greater than 100 acres respectively.

5.0 Energy Dissipation and Shore Protection

A common method of reducing or eliminating wave erosion effects on berms is to construct a protective cover on, or provide sheltering effects along, the berms at the water-line. A major disadvantage of this method is the cost involved. Waste stabilization ponds are intended to provide low cost sewage treatment and the installation of expensive berm protection may render the system uneconomical. A six and one-half acre pond (2.6 Hectares) with a length-to-width ratio of approximately 2:1 will have a berm length of approximately 2200 feet (670m) and it is felt that, if complete protection is to be provided, the maximum cost of



- ① - PROTECTED BEACH
- ② - MODERATELY PROTECTED BEACH
- ③ - EXPOSED BEACH

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BERM SLOPE - GRAIN SIZE
CORRELATION

FIGURE 9

such protection should not exceed \$1.00 per lineal foot.

A further complication arises due to the usual operating procedure of seasonally varying the operating level of the pond between the 3 and 5 foot levels, (1 to 1.7m). Thus, bank protection must be provided over a vertical distance of approximately 3 feet (1m) which, on a 1:4 slope, is equivalent to 13 square feet per lineal foot of berm (4 sq.m/m).

5.1 Energy Dissipators

Floating log booms have been used in one test installation as a method of dissipating wave energy. The logs were of 6 inch nominal diameter (15 cms) and approximately 10 to 12 feet (3-4m) long, chained together end-to-end and anchored to posts driven into the pond bottom in such a position that with a 5 foot liquid depth the log boom floated approximately 4 feet out from the shore line.

This method was tested over a length of approximately 200 feet (70m) at a point in a pond subjected to severe wave attack. After one year, erosion behind the boom was very slight in comparison to severe attack in an unprotected control section.

A disadvantage which became apparent was the growth of weeds behind the boom, due to the lack of wave action. The weeds harbour mosquito larvae and are not considered desirable, even though they provide additional erosion protection.

5.2 Shore Protection

5.2.1 Rip-rap

The use of "man-size" (20-25 kg) cobbles, placed on berm slopes over the entire liquid-depth operating range has been practiced in several installations. If stone is readily available and inexpensive

this is a practical method of shore protection but may not provide protection for an extended period. There is a tendency for the soil under the cobbles to eventually erode and allow the rip-rap to slide down the slope. The proper and careful placement of rip-rap requires a considerable amount of hand labour and may prove uneconomical.

5.2.2 Soil-cement (13)

The use of soil-cement mixtures, consisting of 10-15% Portland cement (by volume of compacted soil-cement) mixed in place with a suitable soil (10-25% passing No. 200 US Std.Sieve) and compacted to 98% maximum density have been successfully used as dam facings and in one waste stabilization pond test installation.

The major draw-back in the use of soil-cement protection, other than the cost, is the lack of suitable soil. It is not economical to bring soil in to the site of berm construction and the available soil may not be suitable for use in a soil-cement mixture. Proper mixing and compaction is necessary.

5.2.3 Sand bagging

The use of jute bags, filled with a mixture of sand and Portland cement (app. 5% by volume) has been successful in preventing erosion in a test installation.

A berm which had been severely eroded was selected for a test site and a wall of sand bags approximately 200 feet long (70m) and 3 bags high was constructed at the beach face. Soil was filled in behind the wall. The operating level of the pond was maintained such that the water level was at approximately the top of the lower bag.

In one year from the time of installation there has been no failure of the wall, or subsequent erosion. Some of the bags have rotted but the sand-cement contents have hardened and remained in position.

This method is effective and is easily installed but its success is dependant upon maintaining a constant liquid level in the pond. Lowering the liquid level below the level of the bags will allow erosion under the wall and subsequent collapse; raising the liquid level above the wall will allow erosion behind the sand bags and again subsequent collapse.

5.3 General

It is felt the following methods would prove effective but to date no test installations have been made. Each method must be evaluated on the basis of the site for which it is proposed.

5.3.1 Plastic film

A thin sheet of polyvinyl may be spread over the berm and anchored. This will be effective if the anchoring is adequate and if the film remains intact. The placement of the sheet is considered critical and difficult.

5.3.2 Snow fencing

A standard snow-fence erected so that the top protrudes above the maximum water level by approximately 6 inches (15 cms) at a distance of 4 feet (1.2m) out from the shore line will dissipate wave energy. Like a log boom, a snow fence will promote weed growths along the shoreline.

5.3.3 Asphalt membrane

An asphalt membrane placed over a soil berm will provide erosion protection as long as the membrane is intact. Deterioration of the entire membrane will be rapid once it is ruptured.

6.0 Suggested Berm Design

6.1 General

The following berm design is presented as an example of the use of the information presented in the previous sections, and the graphs and charts included herewith (Figures 10,11,12).

6.2 Assumed Conditions

Pond size - 7 acres, $L=2W$

Soil - median grain diameter = 0.45mm

Maximum wind velocity - 65 mph

Shelter belt - dense growth, 10
feet high 20 feet
from shoreline

6.3 Calculation

From Fig. 10

maximum fetch = F_1 - 900 feet

From Fig. 2 and 3

$$\begin{aligned} F &= (F_1 + x) - 17a \\ &= (900 + 20) - (17 \times 10) \\ &= \underline{750} \text{ feet} \end{aligned}$$

From Fig. 11

$U = 65$ mph

$F = 750$ feet

$H_s = \underline{1.2}$ feet

GRAPH OF
POND AREA VS. FETCH

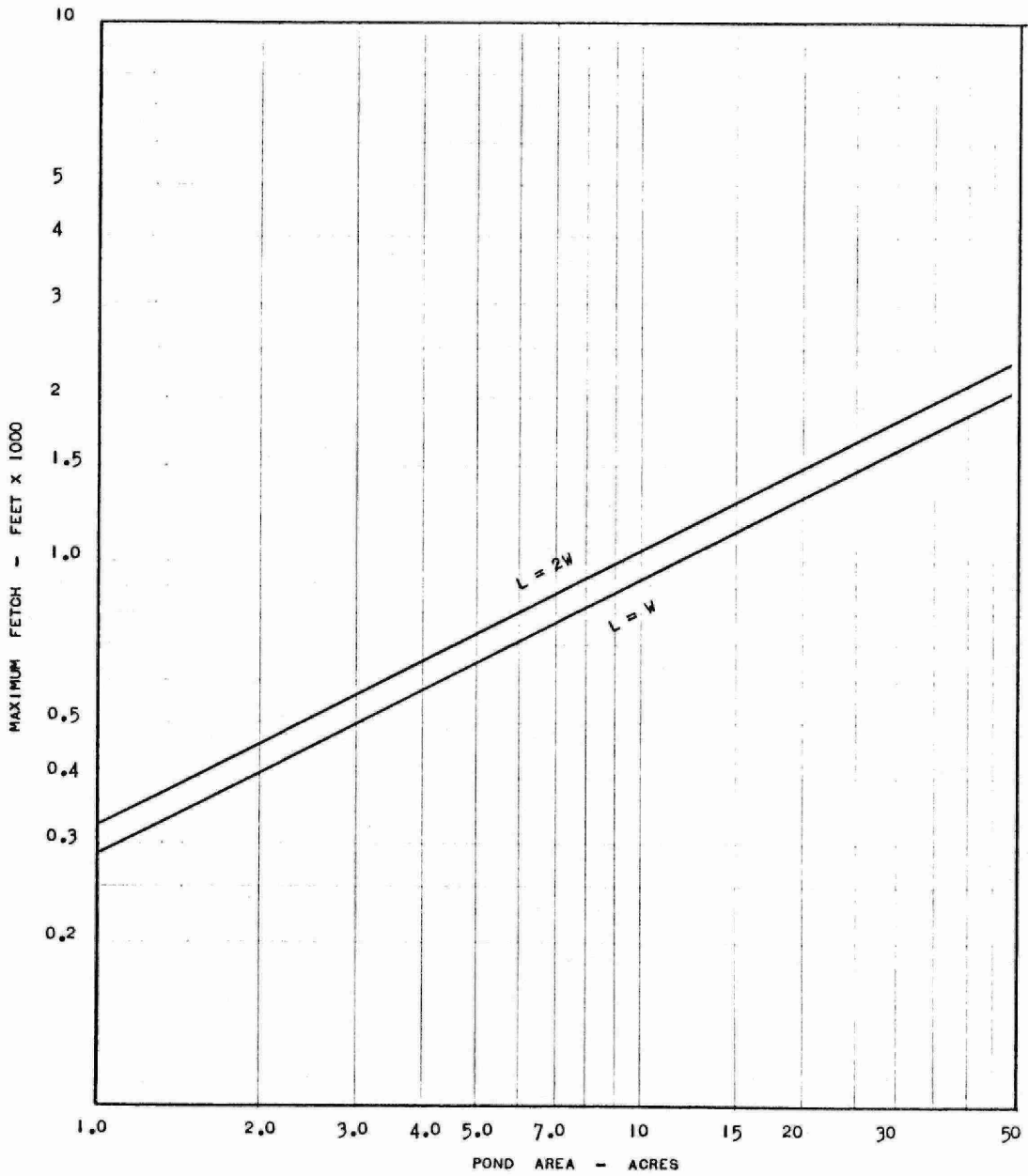


FIGURE 10

-27-

GRAPH OF
POND AREA VS. FETCH

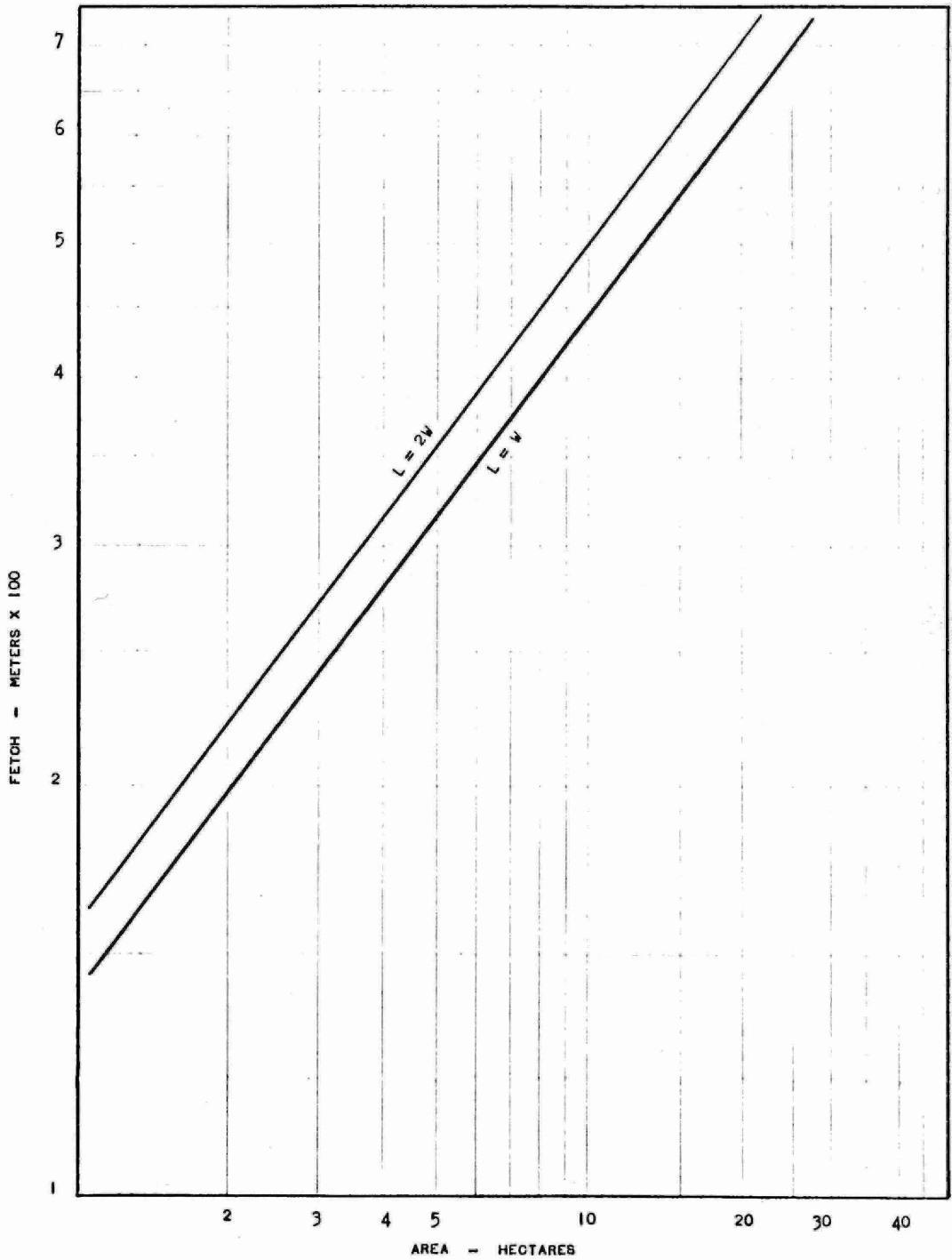


FIGURE 10A

CALCULATION OF
SIGNIFICANT WAVE HEIGHT

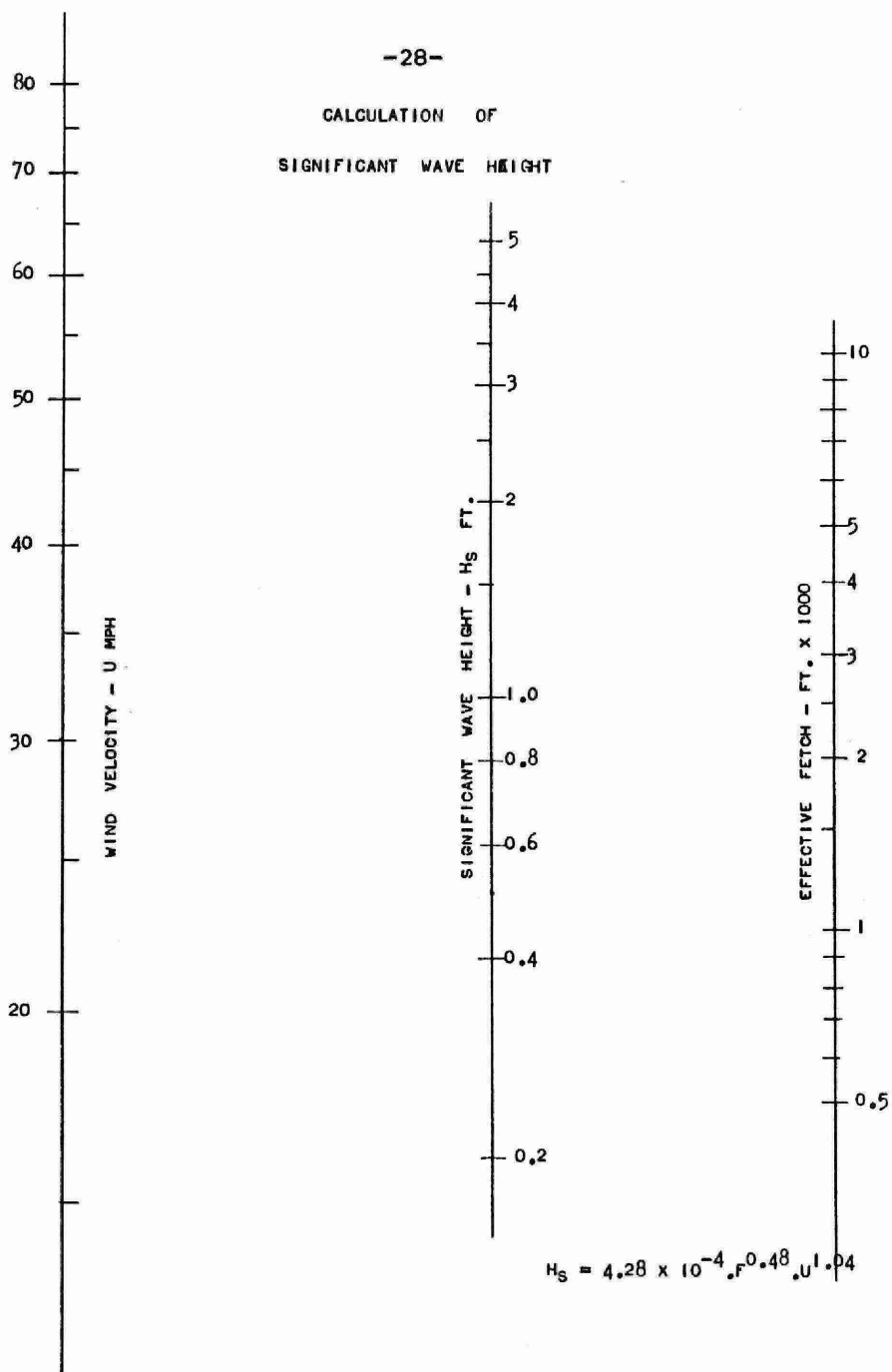


FIGURE 11

CALCULATION OF
SIGNIFICANT WAVE HEIGHT

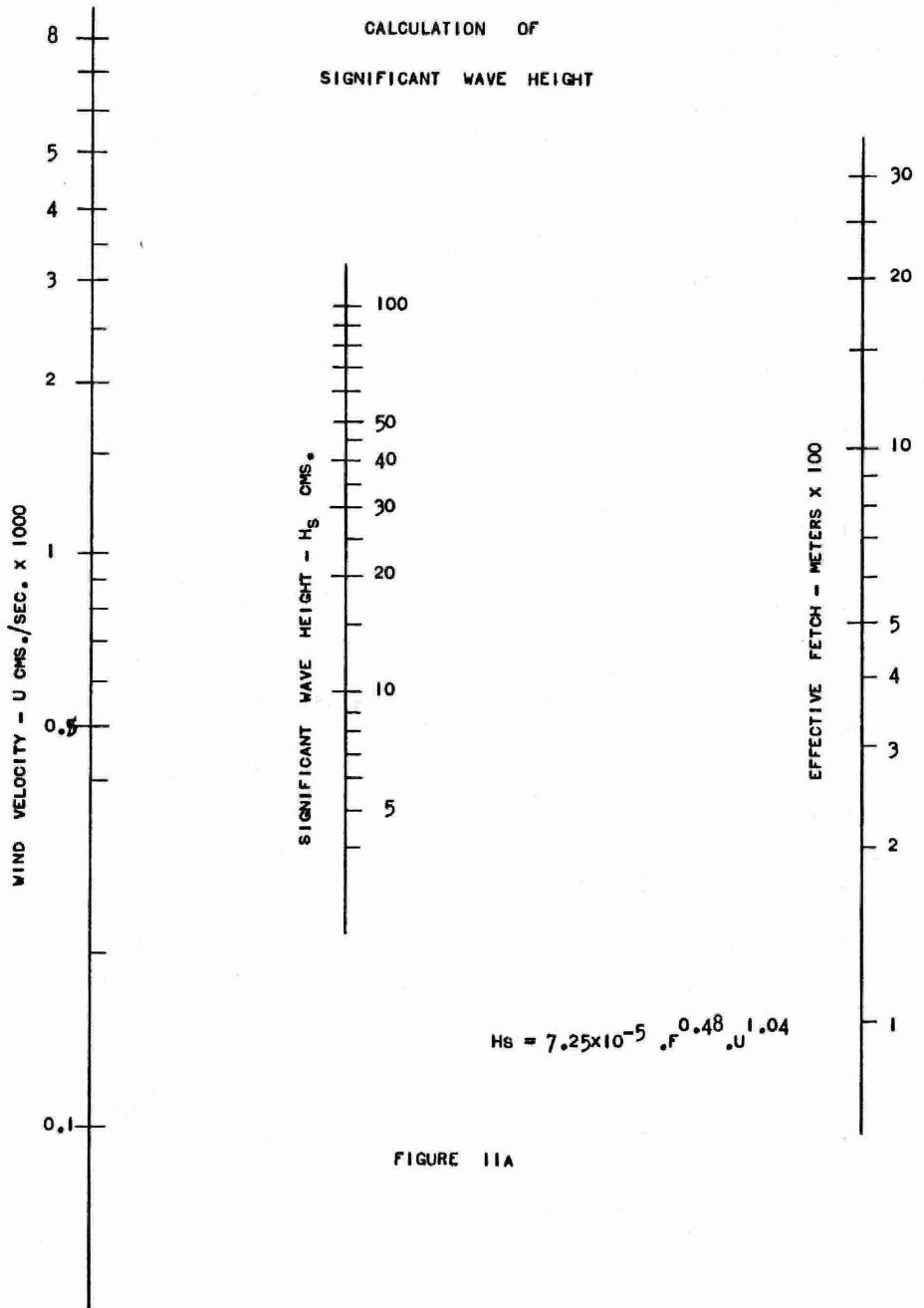


FIGURE 11A

GRAPH OF
WAVE RUN-UP
VS.
EMBANKMENT SLOPE

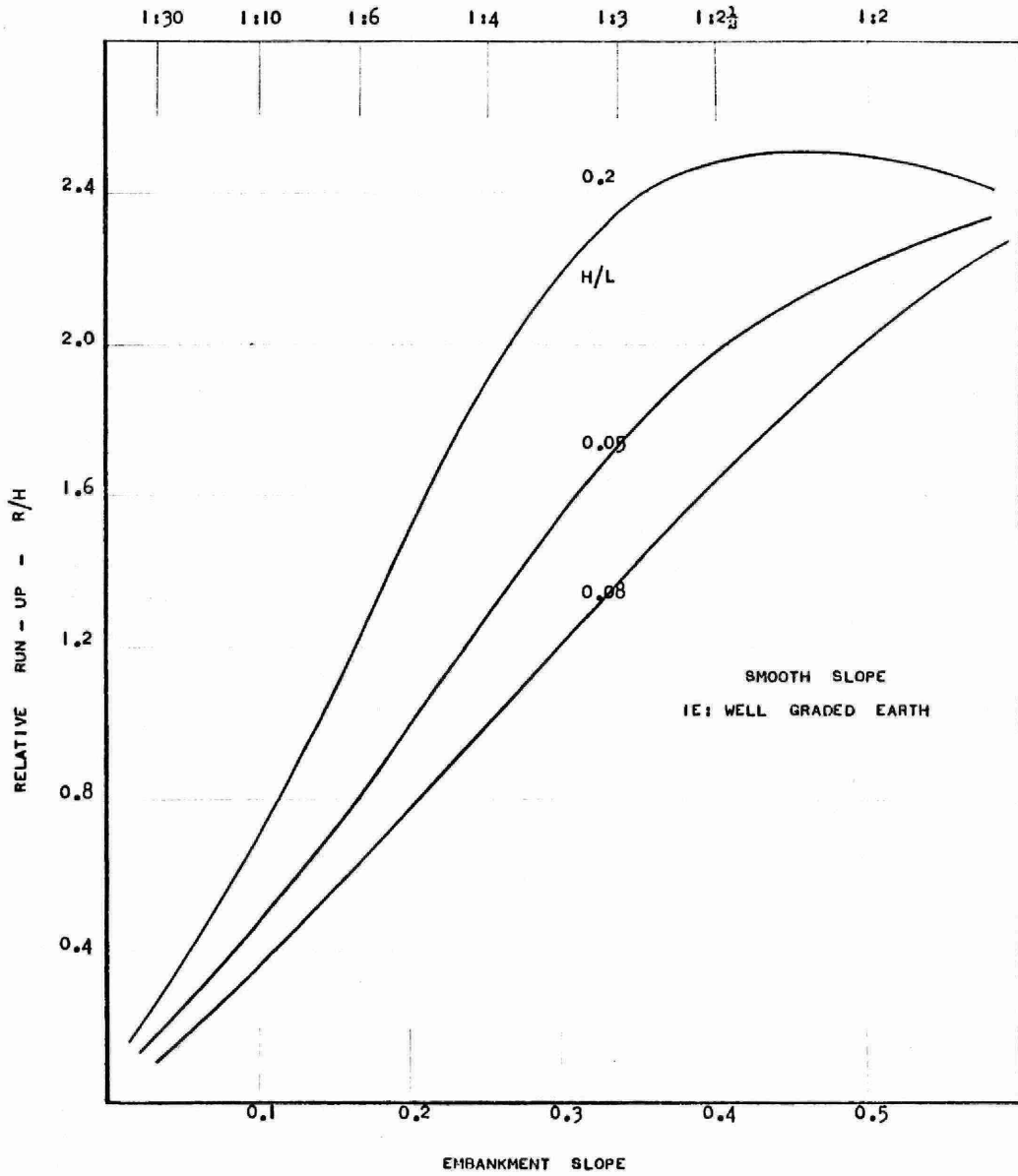


FIGURE 12

From Fig. 9

median grain size = 0.45 mm
protected beach

$$S = \underline{\underline{1:6}}$$

From Fig. 12

$$S = 1:6$$

$$H = 1.2 \text{ feet}$$

$$R = 1.2H \text{ (max)}$$

$$\text{Required freeboard} = 1.44 \text{ feet}$$

Based on the foregoing for a seven acre pond, having a L/W ratio of 2, and with berms built of a material with a median grain size of 0.45 mm., it is recommended that the berms be constructed with an interior slope of 1:6 (vertical;horizontal) and with 1.44 feet of freeboard above maximum operating water-level.

7.0 Conclusions

From the foregoing analysis it is concluded that erosion of waste stabilization ponds, a serious and costly deficiency of this type of waste treatment, may be minimized, if not eliminated, by careful design and construction of the berms, and with the possible additional installation of shore protection and/or wave energy dissipation devices.

In the initial design of the berms a soils analysis is a prerequisite. It is not possible to select arbitrarily an interior slope which will be applicable to all types of soil. It may be evident from a soils analysis that the natural resistance to wave erosion inherent in the soil is so poor that shore protection must be provided. At this time an economics study should be done with one alternative being the selection of another site.

Prediction of wave heights to be experienced in any particular location can be made with reasonable accuracy and the magnitude of waves will provide the data necessary to design the berm freeboard.

The design of a waste stabilization pond should be carefully considered and the design should be followed carefully during construction. The proper design, carefully executed, will provide a waste treatment facility which will require a minimum of maintenance.

8.0 Recommendations

The following recommendations are made on the basis of information obtained from published material and from tests performed in the OWRC laboratory and at operating installations.

1. Prior to design of a waste stabilization pond berm

a soils analysis is required. The interior slope of the berms should be designed on the basis of the soil to be used in constructing the berms.

2. On the basis of predicted maximum significant wave height and wave run-up, (dependant upon berm slope), the required free board may be calculated.

3. If the soil available is of such a nature that a natural slope which is resistant to wave erosion cannot be constructed economically, (Recommended slope not to be shallower than 1:10), and no other suitable site is available, shore protection, and/or wave energy dissipation devices should be provided.

4. The design of shore protection should be made with careful consideration given to the expected operating conditions.

5. Construction of waste stabilization pond berms should be done with careful adherence to design lines, grades, required compaction, etc.

APPENDIX - A

WAVE TESTS

Berm Material - Sand No.2

Berm Slope - 1:2

Wave frequency - 30/min

Test No.	H	R (run-up)	Tf
1	6 cm	11 cms	4 min
2	"		5
3	"		5
4	"		5
5	"		5
6	9 cm		2 min
7	"	-	2
8	10 cm	-	20 sec
9	"		20
10	"		20
11	13 cm	-	15 sec
12	15 cm	-	15

WAVE TESTS

Berm Material - Sand No.2

Berm Slope - 1:3

Wave frequency - 30/min

Test No.	H	R	Tf
101	6 cm	11cm	11 min
102	"	-	11
103	"	11	12
104	"	-	12
105	"	-	11
106	"	-	10
107	8 cm	15	8 min
108	"	-	8
109	10 cm	18	4
110	"	-	5

WAVE TESTS

Berm Material - sand No.2

Berm Slope - 1:4

Wave Frequency - 30/min

Test No.	H	R	Tf
201	6 cm	10 cm	22 min
202	"	-	27
203	"	-	21
204	"	-	22
205	"	-	21
206	"	-	21
207	8 cm	15 cm	5 min

WAVE TESTS

Berm Material - sand No.2

Berm Slope - 1:5

Wave Frequency - 30/min

Test No.	H	R	T _f
301	6 cm	11 cms	4 ^h 30 ^m
302	"	-	6 ^h 30 ^m
303	"	-	5 ^h 30 ^m
304	"	-	5 ^h 30 ^m
305	"	-	4 ^h 30 ^m

Berm Material - sand No.2

Berm Slope - 1:6

Wave Frequency - 30/min

Test No.	H	R	T _f
401	6 cm	9 cm	Very Little erosion*
402	"	"	" "
403	"	"	" "

*Tests terminated at
10 hours

WAVE TEST

Berm Material - L1

Berm Slope - 1:5

Wave Frequency 3-/min

Test No.	H	R	T _f
501	6 cm	9 cm	5 hours*

* Material eroded quickly and then stabilized to some extent. It appeared that the fines (clay sizes) were washed into suspension and remaining material of a fairly uniform grading acted in a manner similar to sand No.2. Large stones (2") helped to dissipate wave energy.

Berm Material - L1

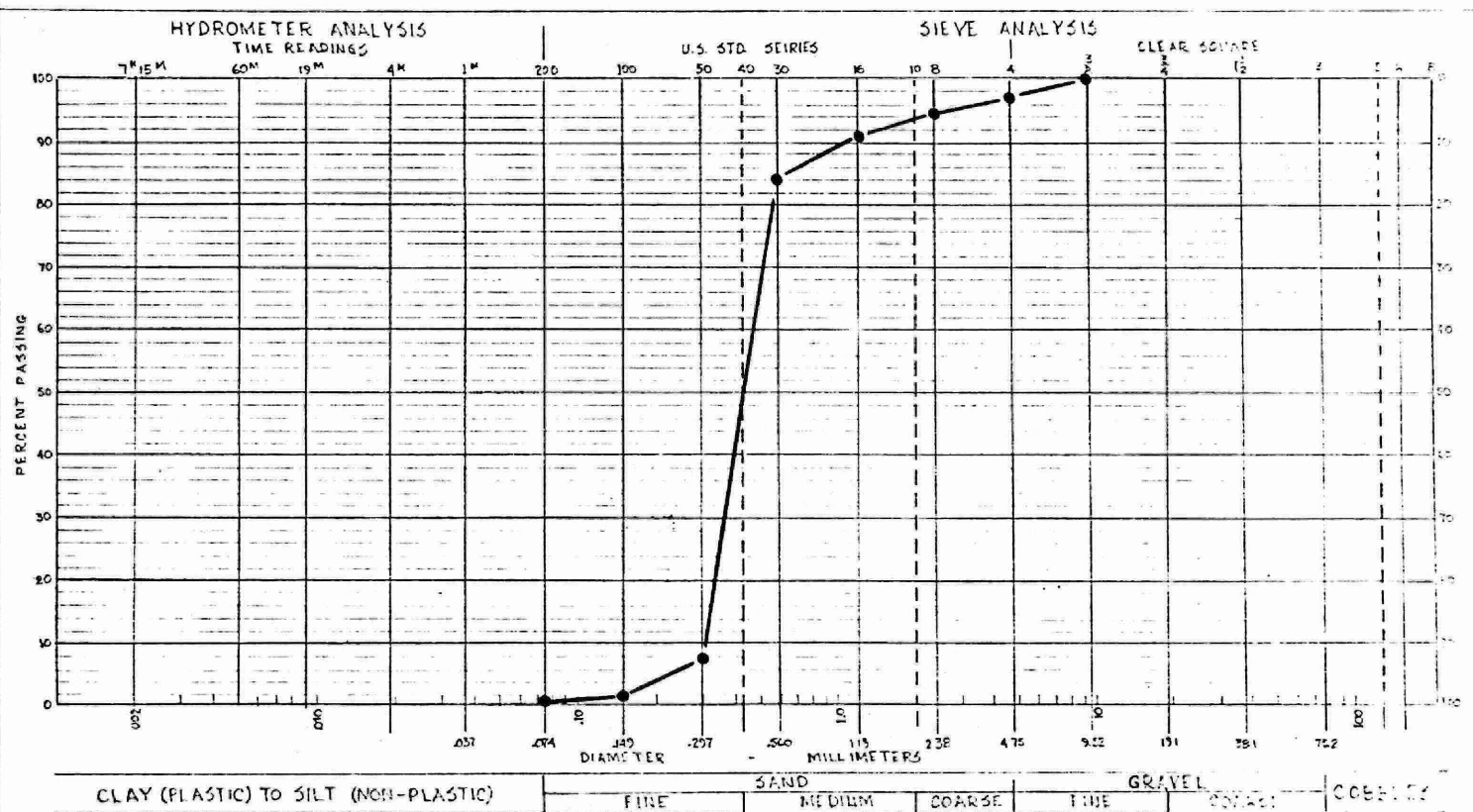
Berm Slope - 1:3

Wave Frequency - 30/min

Test No.	H	R	T _f
601	6 cm		3 min*
602	"		3
603	"		3

* Slope appeared to be the maximum the soil could support as cracks appeared immediately upon wetting. Failure was complete.

APPENDIX - B

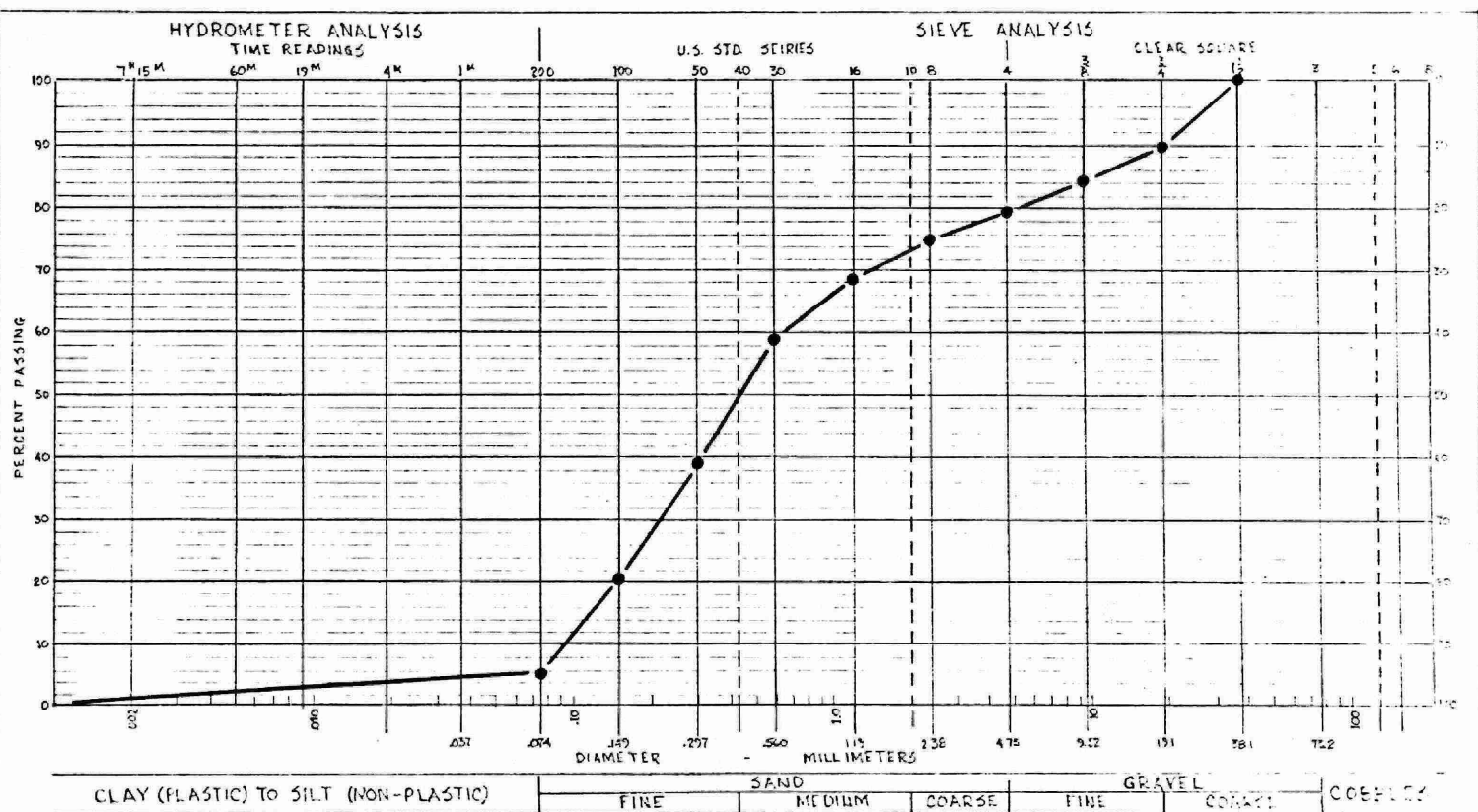


NOTES:

SAMPLE -- SAND N°2 --

DATE Nov 26/65

BY -- *zf* --



NOTES:

SAMPLE - - - L1 - - -

DATE DEC 2/65

BY *[Signature]*

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